WEST WIND LABORATORY
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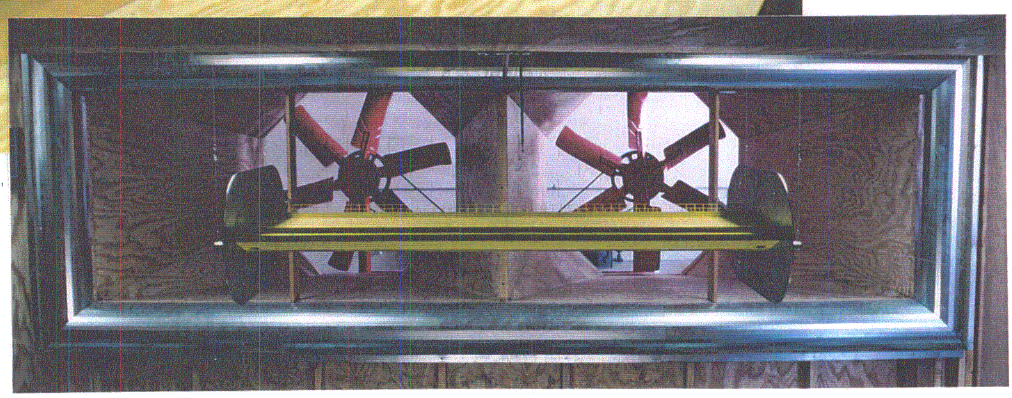
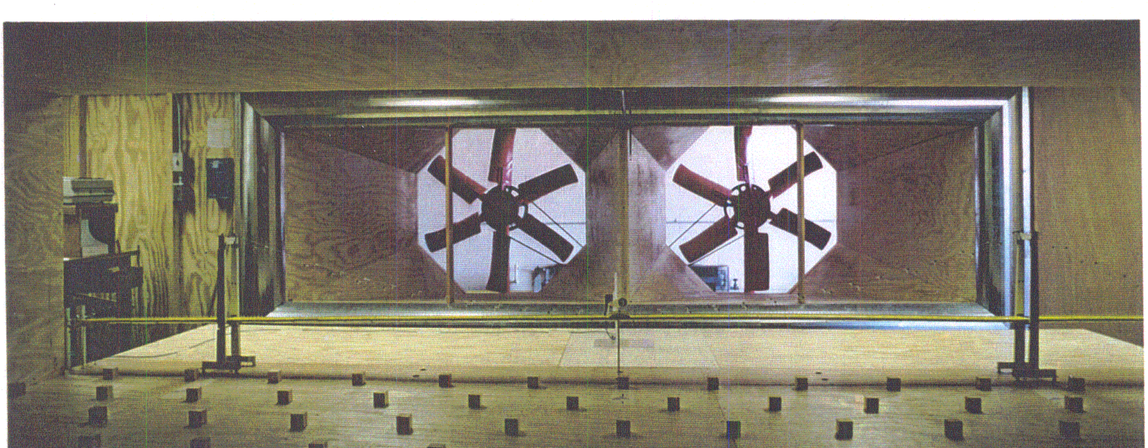
WIND STUDY
THIRD CARQUINEZ STRAIT BRIDGE
CROCKETT TO VALLEJO, CALIFORNIA





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WIND STUDY
THIRD CARQUINEZ STRAIT BRIDGE
CROCKETT TO VALLEJO, CALIFORNIA

for

DE LEUW • OPAC •STEINMAN,
A Joint Venture/Association

by

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West Wind Laboratory, Inc.

Robert H. Scanlan, PhD

Nicholas P. Jones, PhD

9 March 1999

Job No: 97-01

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SUMMARY REPORT

INTRODUCTION AND OBJECTIVES

This report summarizes the results of a wind study performed by the West Wind Laboratory, Inc. and Drs. Robert H. Scanlan and Nicholas P. Jones to determine the performance of the proposed 3rd Carquinez Straits Bridge between Vallejo and Crockett, California. The objectives of the study were as follows:

1) Determine design wind speed and critical flutter wind speed threshold;

2) Determine the critical flutter wind speed for the proposed bridge;

3) Determine the dynamic buffeting response of the bridge to design level, naturally occurring turbulent winds;

4) Determine the responses of a free-standing tower (during the construction phase) to naturally occurring turbulent winds; and

5) Determine static equivalent drag coefficients for various components of the bridge.

The West Wind Laboratory, Inc. (WWL) performed all experimental wind tunnel testing for this study. The WWL performed the section model tests, the full bridge model tests, and the tests of the free-standing tower that would occur in the construction phase only. Drs. Scanlan and Jones determined, mathematically, the critical flutter wind speed and buffeting response of the full, final completed bridge configuration only, using the data generated by the WWL. The WWL and Drs. Scanlan and Jones developed the design wind speed criteria, and prepared this summary report.

This report is presented in three parts: 1) this summary, 2) those portions of the study for which Drs. Scanlan and Jones were responsible, and 3) those portions for which the WWL was responsible.

The following definitions are used in this report:

A "design wind speed" is the wind speed criterion to use for the service level design of the bridge. This is a 100 year recurrence, one-hour averaged wind speed, at an elevation of 50 m above the water surface.

A "critical flutter wind speed threshold" is the minimum acceptable wind speed at which aeroelastic flutter can occur and still maintain all design factors of safety with respect to the loads produced by the design wind speed. This is a 10-minute averaged wind speed at an elevation of 50 m above the water surface, and in this study is a wind speed that has a recurrence period of 10,000 years.

A "flutter wind speed" is the actual 10-minute averaged wind speed, at an elevation of 50 m above the water surface, at which the bridge flutters, for a given wind direction, angle of incidence, mode of vibration, and/or combination of modes of vibration.

A "critical flutter wind speed" is the lowest "flutter wind speed" for all possible cases.

The "railing solids ratio" is defined as the ratio of the area of the solid railing elements, above the curb, to the area of the total railing, above the curb, projected on a vertical plane parallel to the bridge deck axis.

DESIGN WIND SPEED CRITERIA

The 100-year recurrence, one-hour averaged, design wind speed at an elevation of 50 m above the water surface was determined to be

$$U = 47.03 \text{ m/sec } (105.21 \text{ mph})$$

The 10-minute averaged critical flutter wind speed threshold, at an elevation of 50 m above the water surface, was determined to be

$$U_{CRIT} = 70.15 \text{ m/sec } (156.93 \text{ mph})$$

See Chapter 2, Ref. B for a more detailed description of the design wind speed criteria.

CRITICAL FLUTTER WIND SPEED

Flutter wind speeds were obtained for winds from the East and West (perpendicular to the bridge deck axis), for vertical angles of incidence of -5, -2.5, 0, 2.5, and 5 degrees. All flutter wind speeds were obtained with a bicycle railing that has a 12% railing solids ratio. The critical flutter wind speed at the bridge deck level was determined to be 72.87 m/sec (163 mph) for winds from the East, with a positive angle of incidence of 5 degrees. Single degree of freedom torsional motion in Mode 14, in smooth flow, with a mechanical damping ratio of 0.3% was found to be critical. The critical flutter wind speed in naturally occurring turbulent winds is expected to exceed 72.87 m/sec (which already exceeds the design threshold of 70.15 m/sec).

See Section II, Ref. A for a more detailed description of the critical flutter wind speed analysis.

BUFFETING RESPONSE

The dynamic buffeting response of the bridge to naturally occurring winds with a one hour averaged wind speed of 38.0 m/sec (85 mph) at an elevation of 50 m are given in Figures S1, S2, and S3 for West winds, and in Figures S4, S5, and S6 for East winds. It was assumed that all modes had a mechanical damping ratio of 0.3%.

See Sections III and IV, Ref. A for a more detailed description of these results, for results with other damping ratios, and for results with other wind speeds up to the design one hour averaged wind speed of 47.03 m/sec (105.21 mph).

BUFFETING RESPONSE OF A FREE-STANDING TOWER

The towers are particularly vulnerable to wind loads when they are free standing, during the construction phase, and before they are braced in the NS direction by the main suspension cables.

Free-standing tower top displacements for NS winds and EW winds are shown in Figures S7 and S8.

Of particular interest is the very large, vortex induced response of the tower in the NS direction, due to winds from the EW direction, at a one hour averaged wind speed of 12.67 m/sec (28.34 mph) at an elevation of 50 m above the water surface. This is of particular concern because winds of this strength, from this direction, will occur regularly every afternoon in Spring, Summer, and Fall when the Central Valley becomes very warm.

It is recommended that aerodynamic or mechanical modifications (temporary modifications) be employed in the tower design to mitigate this potential problem during the construction phase, when the towers are free standing. Vortex induced tower motions will not occur in the final configuration, after the main suspension cables have been installed.

See Chapter 6, Ref. B, for a more complete description of the free-standing tower tests and results.

MISCELLANEOUS STATIC AERODYNAMIC ELEMENTAL CHARACTERISTICS

Drag coefficients of 1.0 and 1.3 were measured for NS and EW winds respectively on an individual tower leg.

A static drag coefficient of 0.166 (with respect to a reference length of B = 27.2 m) was measured for horizontal transverse winds on the bridge. A drag coefficient of 0.0332 (20% of 0.166) was computed as being appropriate for longitudinal wind loading on the deck. For winds at an angle of φ off of the EW axis, the aerodynamic loading on the bridge deck should be

$$F(\varphi) = ((F_T Cos\varphi)^2 + (F_L Sin\varphi)^2)^{1/2}$$

where

F_T total transverse wind load; and

F_L total longitudinal wind load.

These drag coefficients should be used with one-hour averaged wind speeds and a gust factor of at least 1.9, for overall horizontal wind loading on the bridge deck and towers *only*. This is an approximation to the results obtained from the detailed buffeting analysis. The approximate gust factor approach must include all of the extremes of the detailed buffeting analysis for it to be valid. There will be different gust factors for different stresses and member actions. Where there are discrepancies between the two approaches, results from the detailed buffeting analysis always take precedence over results from an approximate gust factor approach.

See Chapter 7, Ref. B for a more detailed description of these miscellaneous static aerodynamic coefficients.

EVOLUTION OF THE DESIGN (RESULTS FROM THE PRELIMINARY STUDY)

Preliminary section model studies were performed on the proposed (35% submittal) bridge deck section. The dimensions and details of the final bridge deck design differed from the preliminary design, but were close enough so that results from comparative studies using the preliminary design would be representative of results for the final design.

The objectives of the preliminary study were to make a quick evaluation, with respect to aeroelastic flutter stability, of the proposed design, and to propose modifications to the proposed design should its performance in strong winds be unacceptable.

The originally proposed section differed only slightly from the final design studied. The overall width, width of sloped portions on the underside and location of the service rails all changed slightly. The original pedestrian railing on the west side of the walkway, had a railing solids ratio of 28%. With this railing the bridge was torsionally unstable, for West winds at 34.24 m/sec (76.59 mph), well below the critical flutter threshold wind speed of 70.15 m/sec (156.93 mph).

The railing was identified as the principle cause of the torsional instability. The preliminary section model tests were performed for sections with railing solids ratios of 0, 12, 14, and 28 percent. The A_2^* curves for those cases are shown in Figure S9.

The curves are for West winds only. Torsional flutter wind speeds (for an assumed mechanical damping of 0.3%) occur (at a value of U/nB) where the A_2^* curve intersects a horizontal line equal to 0.018. This is an approximation, but an adequate approximation to evaluate relative effects of leading edge, bicycle railing solids ratios. From the curves, it was determined that the torsional flutter wind speeds were 34.24 m/sec (76.59 mph) for a solids ratio of 28%; 41.08 m/sec (91.89 mph), for 14%; infinite (or beyond all winds included in this study (approximately 250 mph)), for 12%; and infinite, for 0%.

Flutter wind speeds were not sensitive to railing solids ratios for East winds. The railing solids ratio is important only when the railing is on the leading edge.

If it is assumed that the A_2^* curve for a bicycle railing solids ratio of 13% was exactly halfway between that for 12% and 14%, its maximum value would be about 0.005, well below the value of 0.018 at which torsional flutter would occur. The flutter speed, for horizontal winds from the West, for a bridge deck with a bicycle railing with a solids ratio of 13% would then also be infinite (or again, beyond about 250 mph).

From these preliminary studies, it was recommended that the railing on the final design have a railing solids ratio of 12%. All final tests, and all results presented in this final report (Part A and B), were made with a railing solids ratio of 12%. For West winds, for vertical angles of attack from -5 degrees to +5 degrees, no torsional flutter instabilities were observed (up to at least 111.76 m/sec (250 mph)). For these cases the A_2^* curves were slightly more negative than the corresponding curves from the preliminary study, i.e., the final shape was slightly more stable than the preliminary shape. For East winds, the only flutter wind speed found was 72.87 m/sec (163 mph) for a positive vertical angle of attack of 5 degrees. The critical flutter wind speed found was therefore 72.87 m/sec (163 mph), for the railing with a railing solids ratio of 12%.

Since final A_2^* curves were better than preliminary A_2^* curves (for the 12% solids case), it is reasonable to assume that the final A_2^* curves would be better for the 13% solids case than they were estimated to be from the preliminary study. Since the deck with a 13% solids ratio was shown to be torsionally stable (up to 111.76 m/sec (250 mph) at least) using preliminary data, for West winds, it is reasonable to assume that it would also be stable for the final configuration. Since the flutter velocity did not appear to be sensitive to the railing solids ratio for East winds (when the railing is on the trailing edge), it is reasonable to assume that the flutter wind speed for East winds, with a positive angle of attack of 5 degrees, would also be 72.87 mph (163 mph). Therefore, the critical flutter wind speed for the bridge with a railing with a railing solids ratio of 13% is also expected to be 72.87 mph (163 mph).

Torsional instability of the bridge was found to be sensitive to the railing solids ratio of the bicycle railing (perpendicular to the axis of the bridge). As the solids ratio increases, the railing becomes more opaque, the bridge is no longer streamlined, suctions develop behind the railing creating leading edge static lift coefficients that deviate significantly from that with an open railing. (Leading edge aerodynamic characteristics dominate the torsional stability of bridge decks. If the leading edge static lift coefficient slope is airfoil-like, torsional velocities are damped, and torsional motions are generally stable. If the leading edge is bluff and girder-like perpendicular to the wind, the static lift coefficient slope may in fact be negative, which will lead to aerodynamic moments that reinforce torsional velocities, that can lead to a torsional instability). No matter how solid the railing may appear to non-perpendicular winds, if the railing is open perpendicular to the bridge axis, the horizontal component of those winds will still flow smoothly over the deck, the deck will appear streamlined to the winds that destabilize the bridge, and suctions will not develop in the lee of the railing. Furthermore, for non-perpendicular winds, the flutter speed will not be critical because the perpendicular component is greatest when winds are, of course, perpendicular.

The detailed final report that follows (Part A and B) investigates the performance of the final bridge design with a railing solids ratio of 12%. In August 1998, the railing solids ratio was changed from 12% to 13%. As argued above, the critical flutter wind speed for this case is expected to be 72.87 m/sec (163 mph), the same as the case with the railing solids ratio of 12%.

REFERENCES

A. Scanlan, R. H. and Jones, N. P., *Wind Response Study Carquinez Strait Suspended Span*, Report for West Wind Laboratory, Inc. and OPAC Consulting Engineers, November 30, 1998.

B. Raggett, J. D., *Experimental Test Results Wind Study Third Carquinez Strait Bridge, Crockett to Vallejo, California,* West Wind Laboratory, Inc., November 1998.

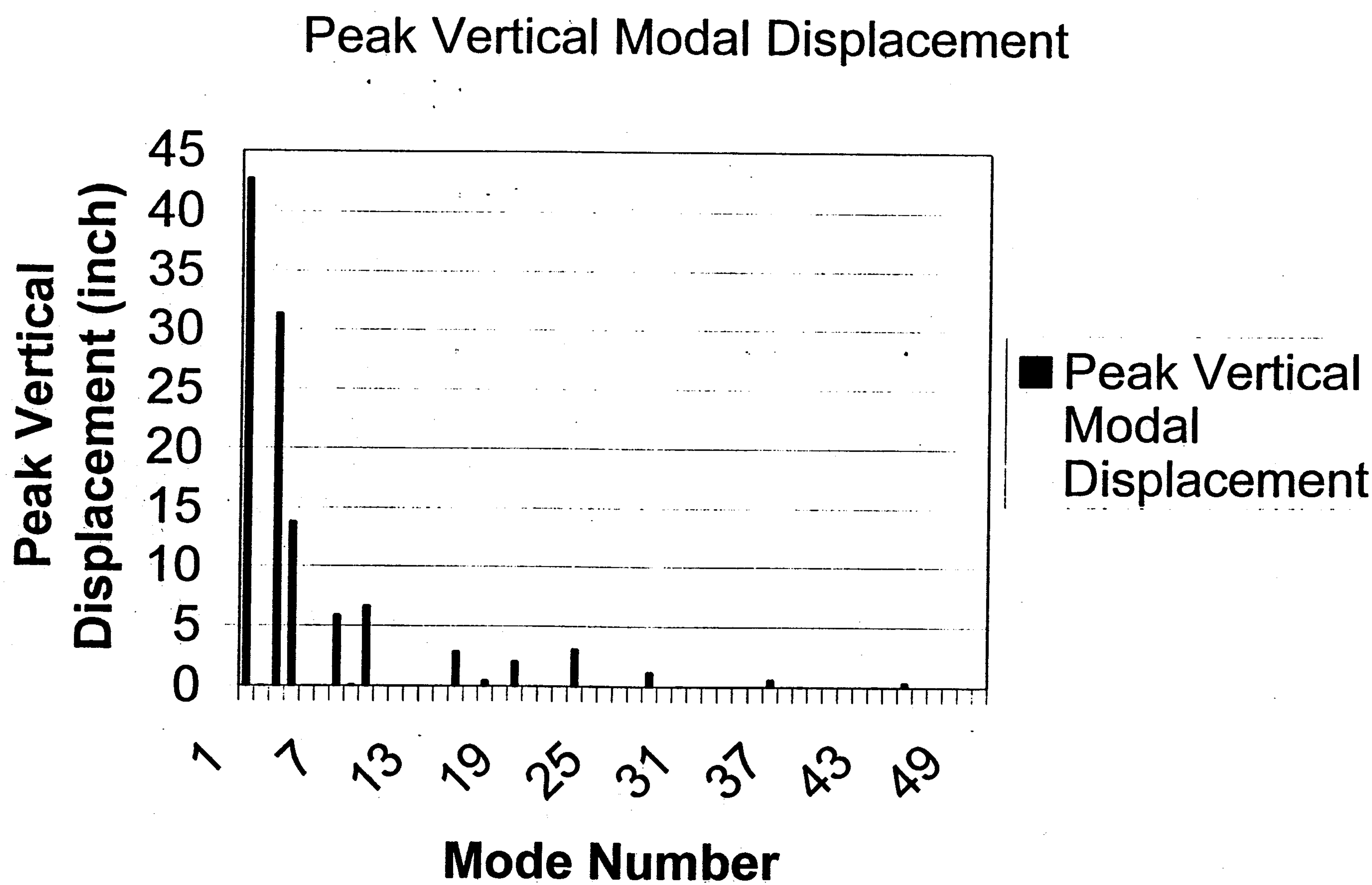

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FIGURE S1
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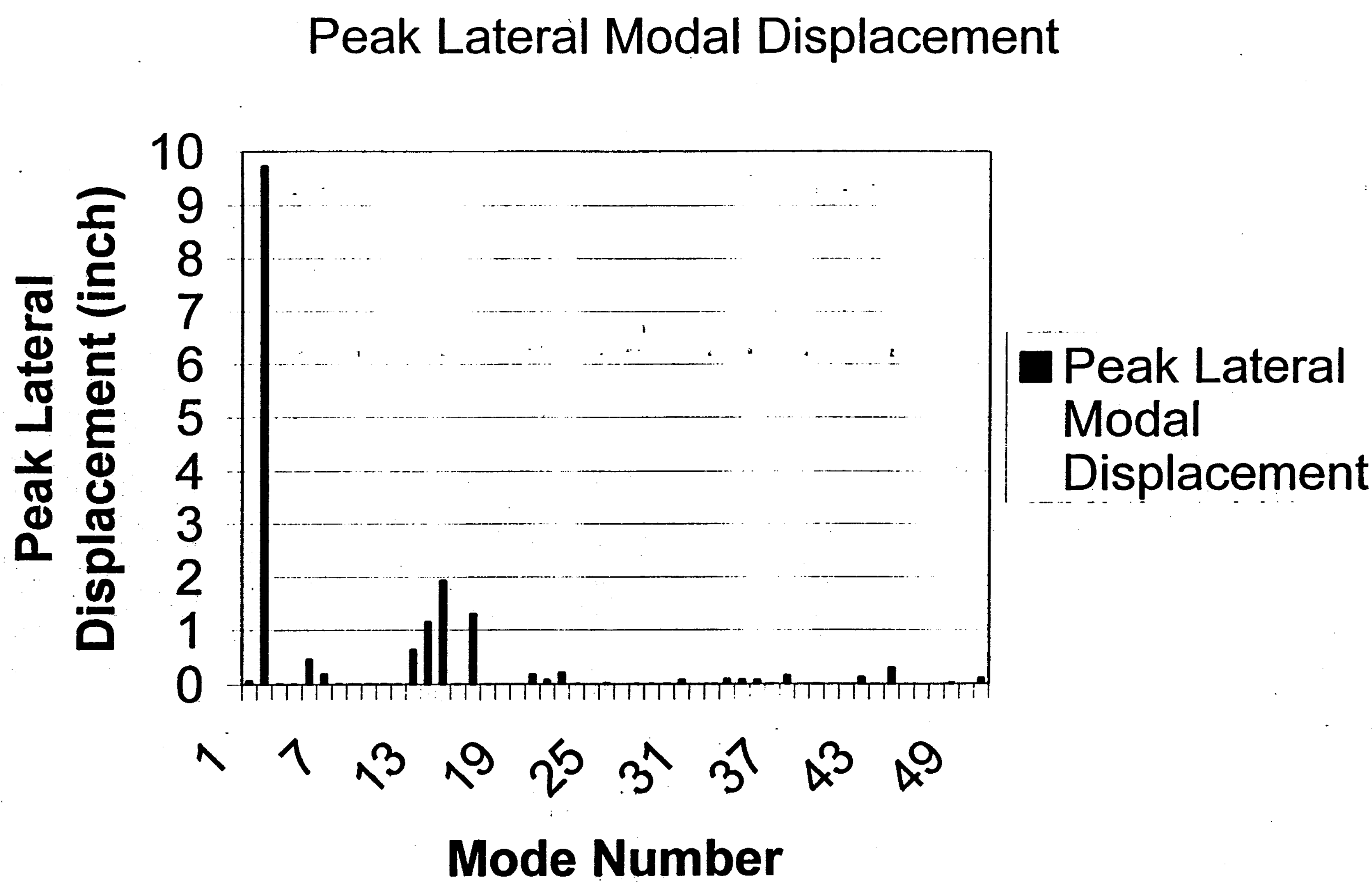

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FIGURE S2
WEST WIND

Peak Torsional Modal Displacement

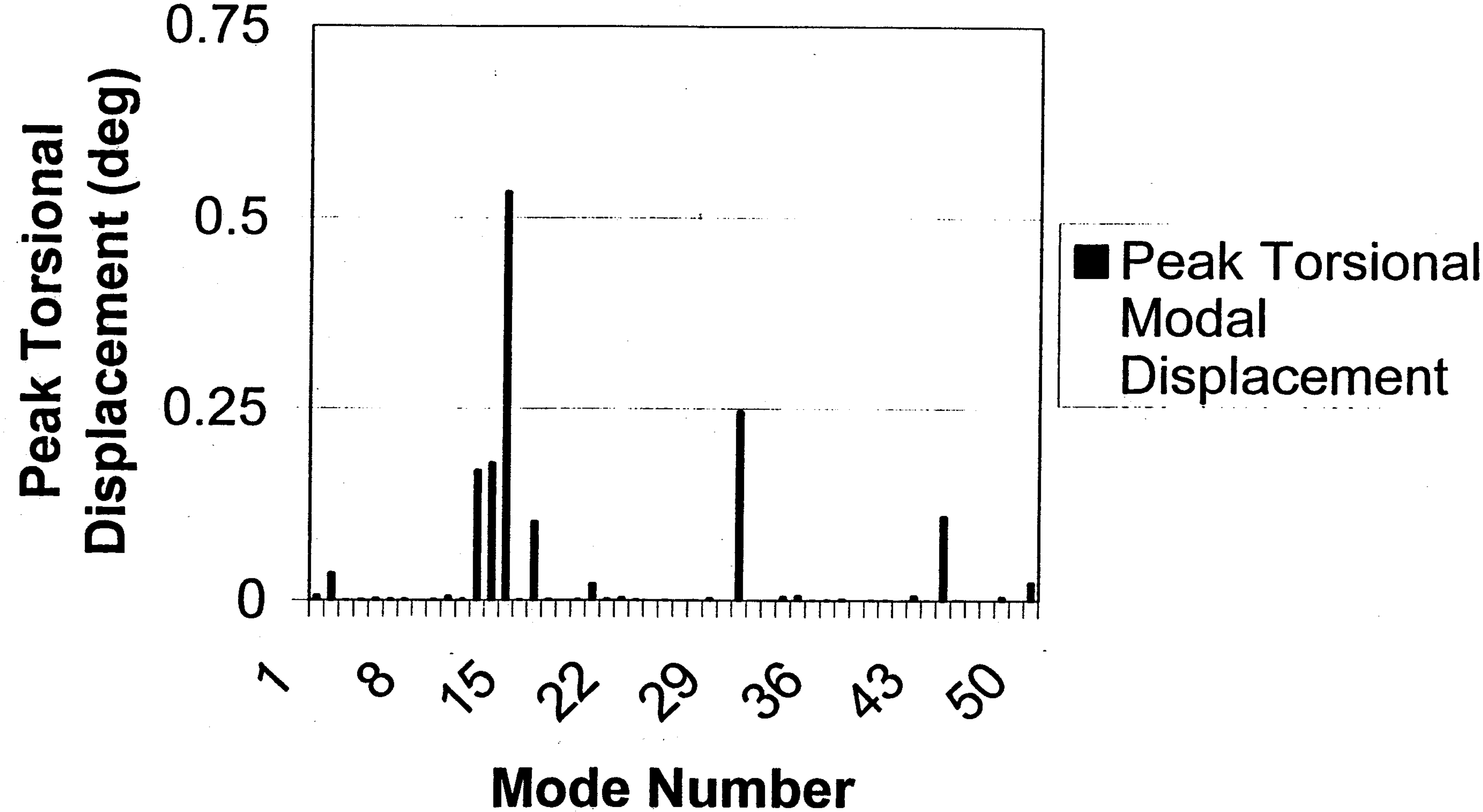

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FIGURE S3
WEST WIND

Peak Vertical Modal Displacement

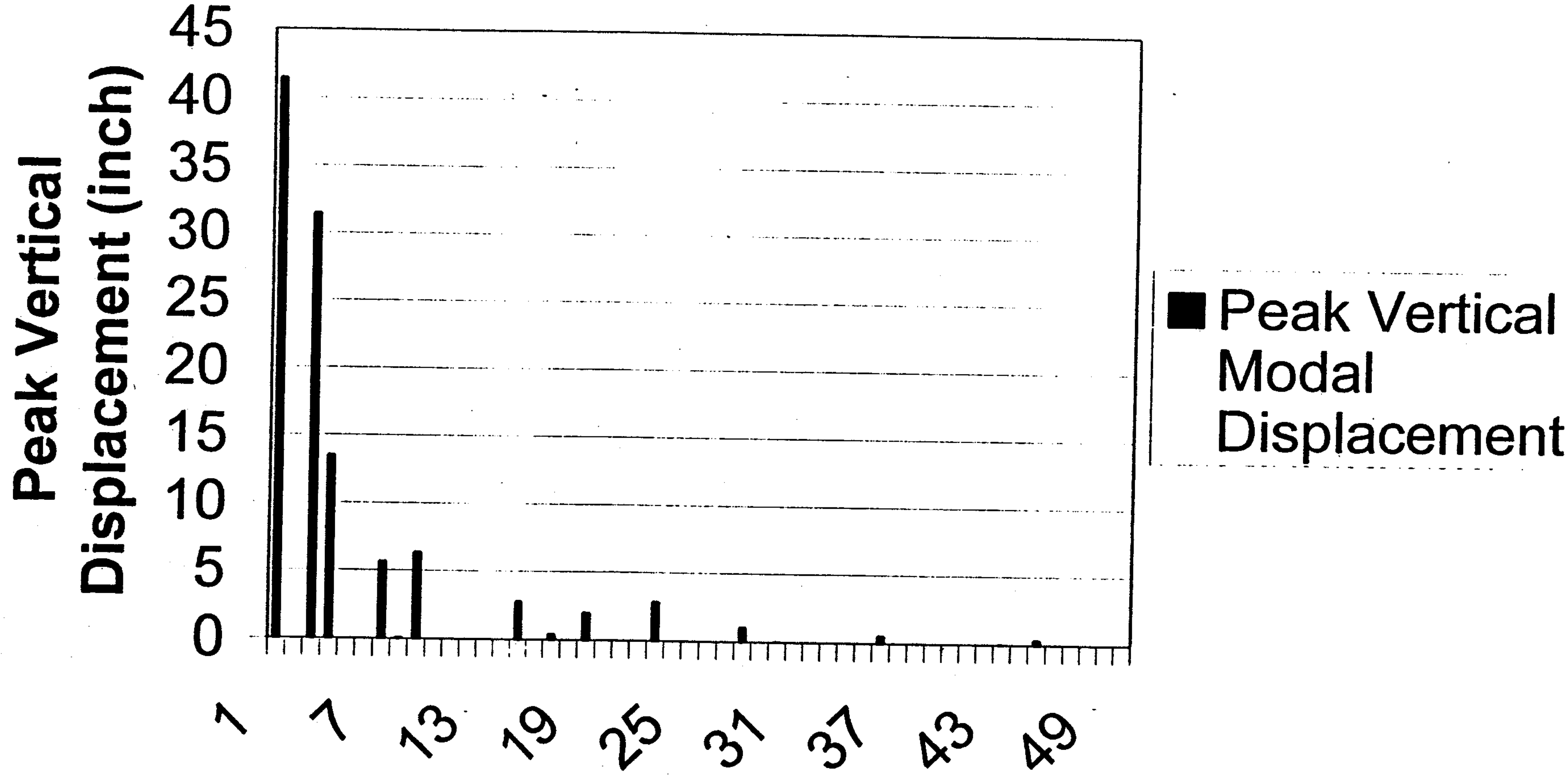

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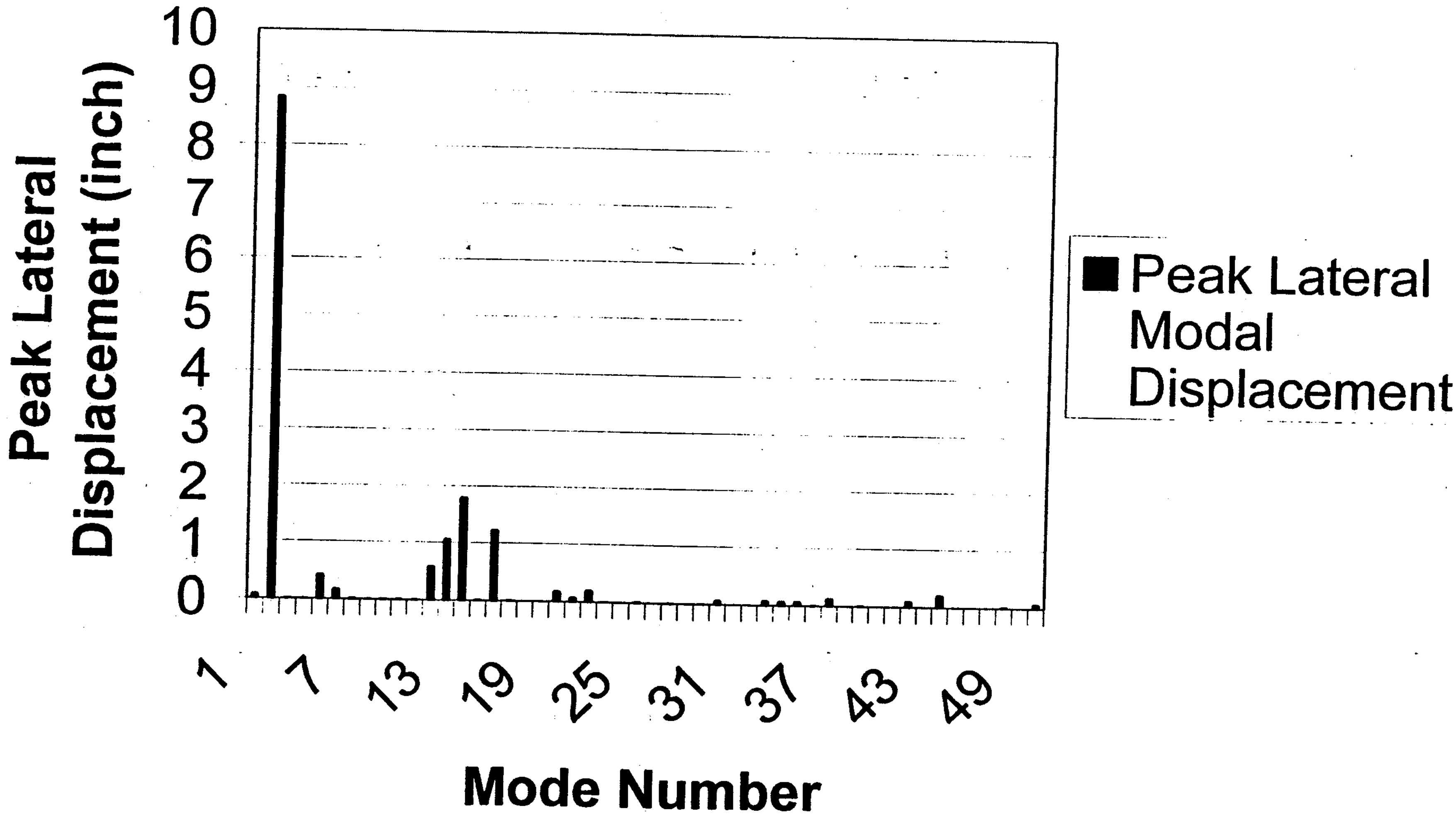
FIGURE S4
EAST WIND

Peak Lateral Modal Displacement



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FIUGRE S5
EAST WIND

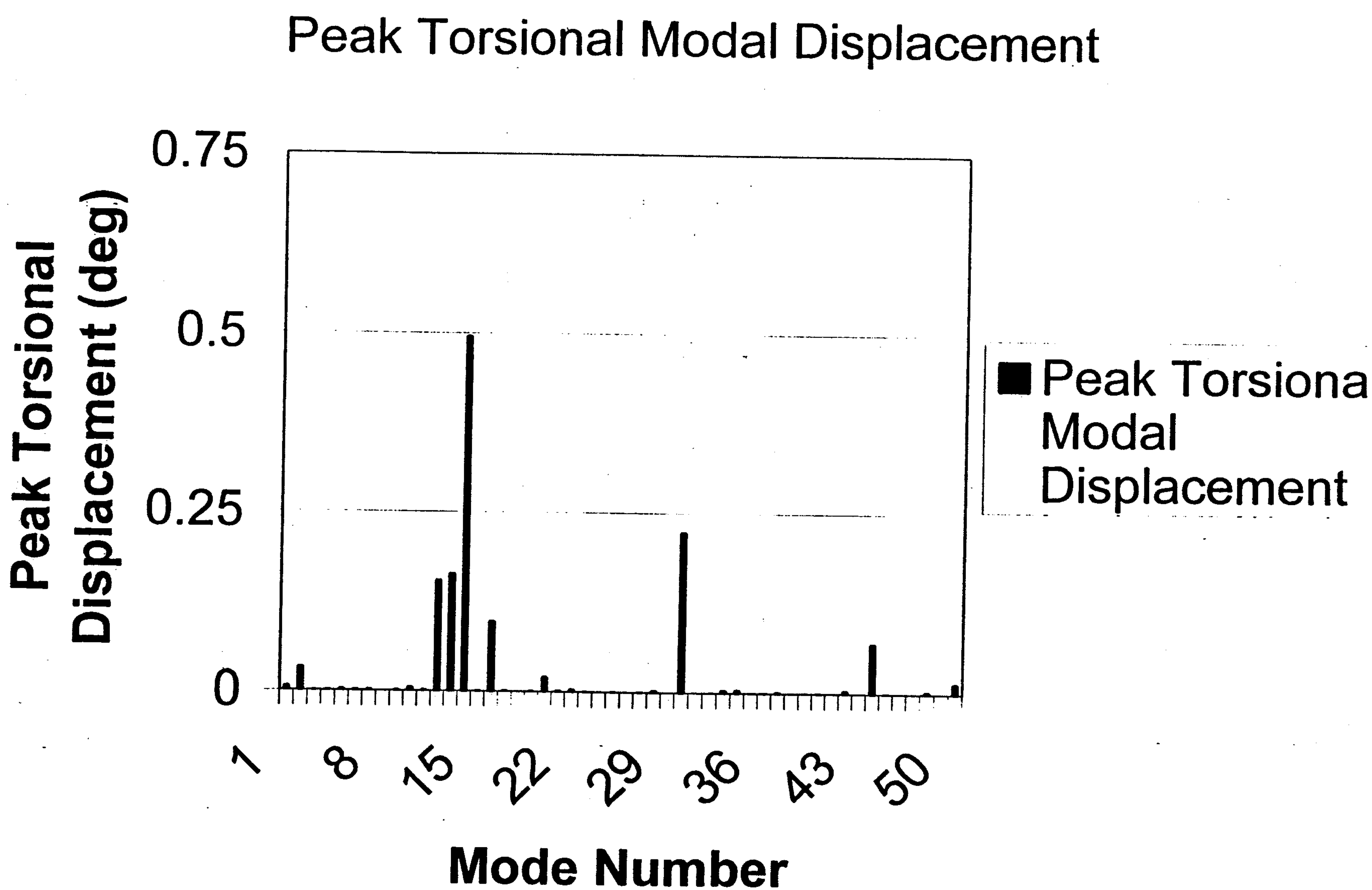

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FIGURE S6
EAST WIND

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CARQUINEZ STRAITS BRIDGE
AZIMUTH ANGLE 0

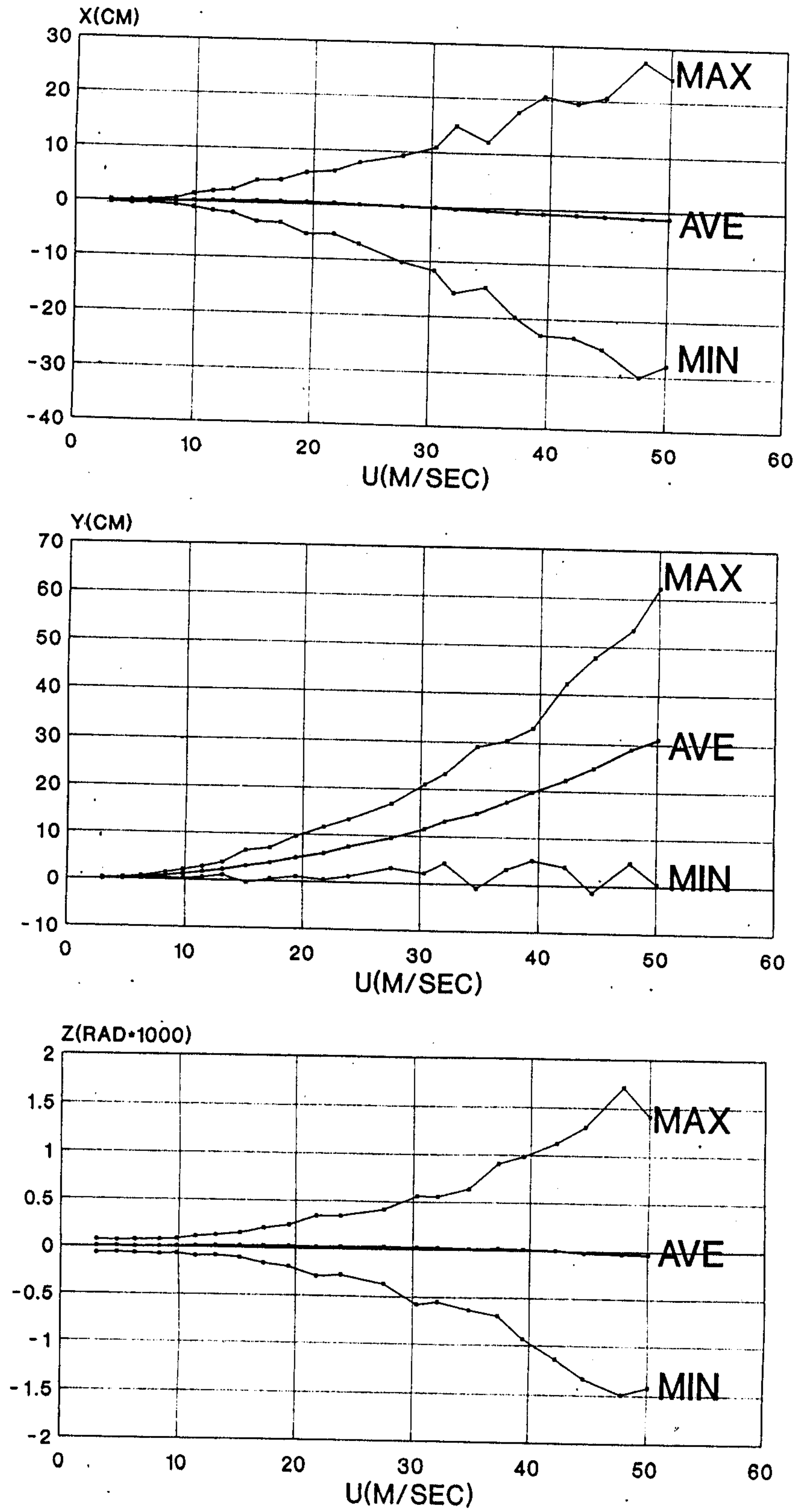

FIGURE S7

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CARQUINEZ STRAITS BRIDGE
AZIMUTH ANGLE 90

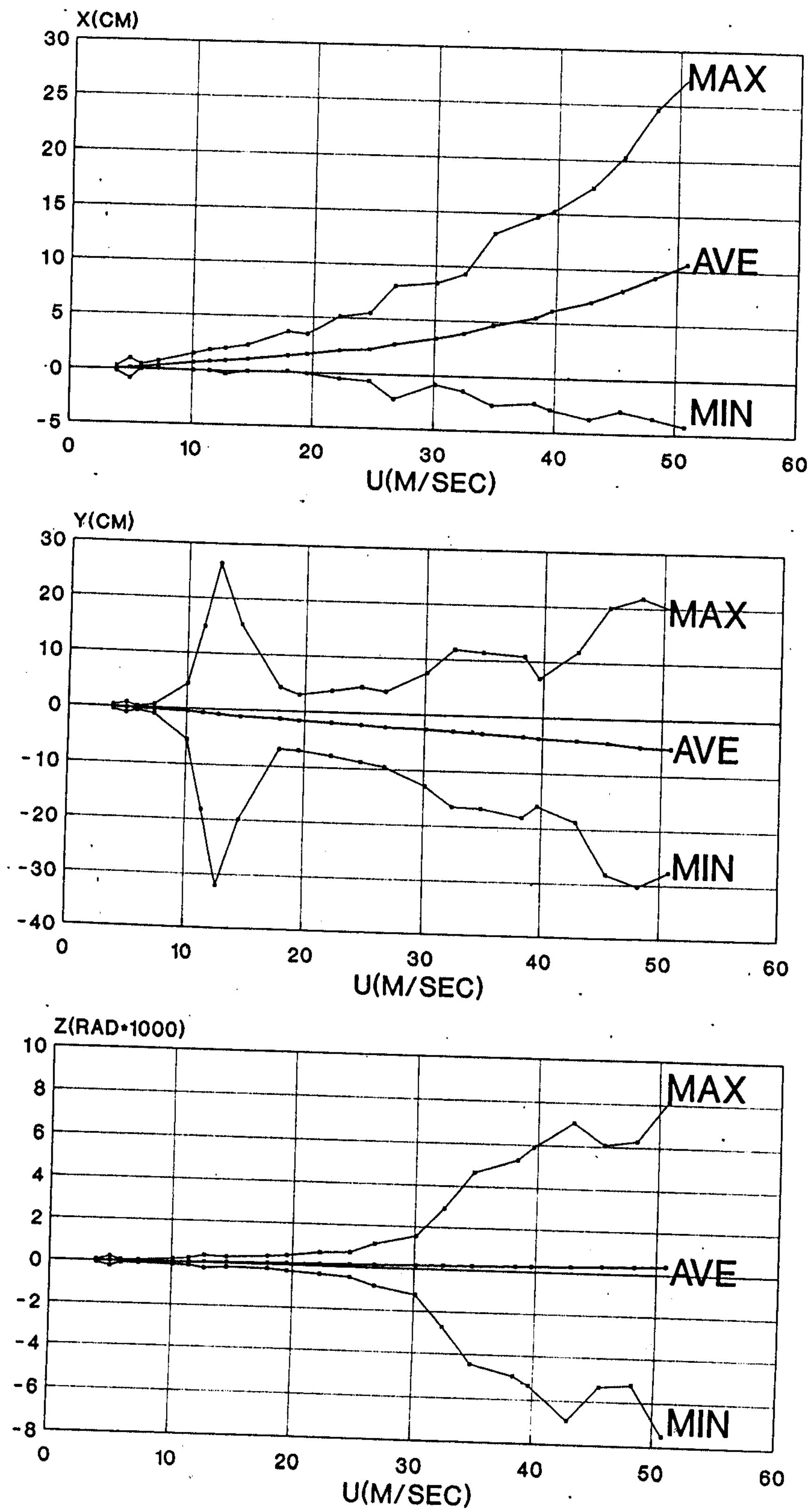
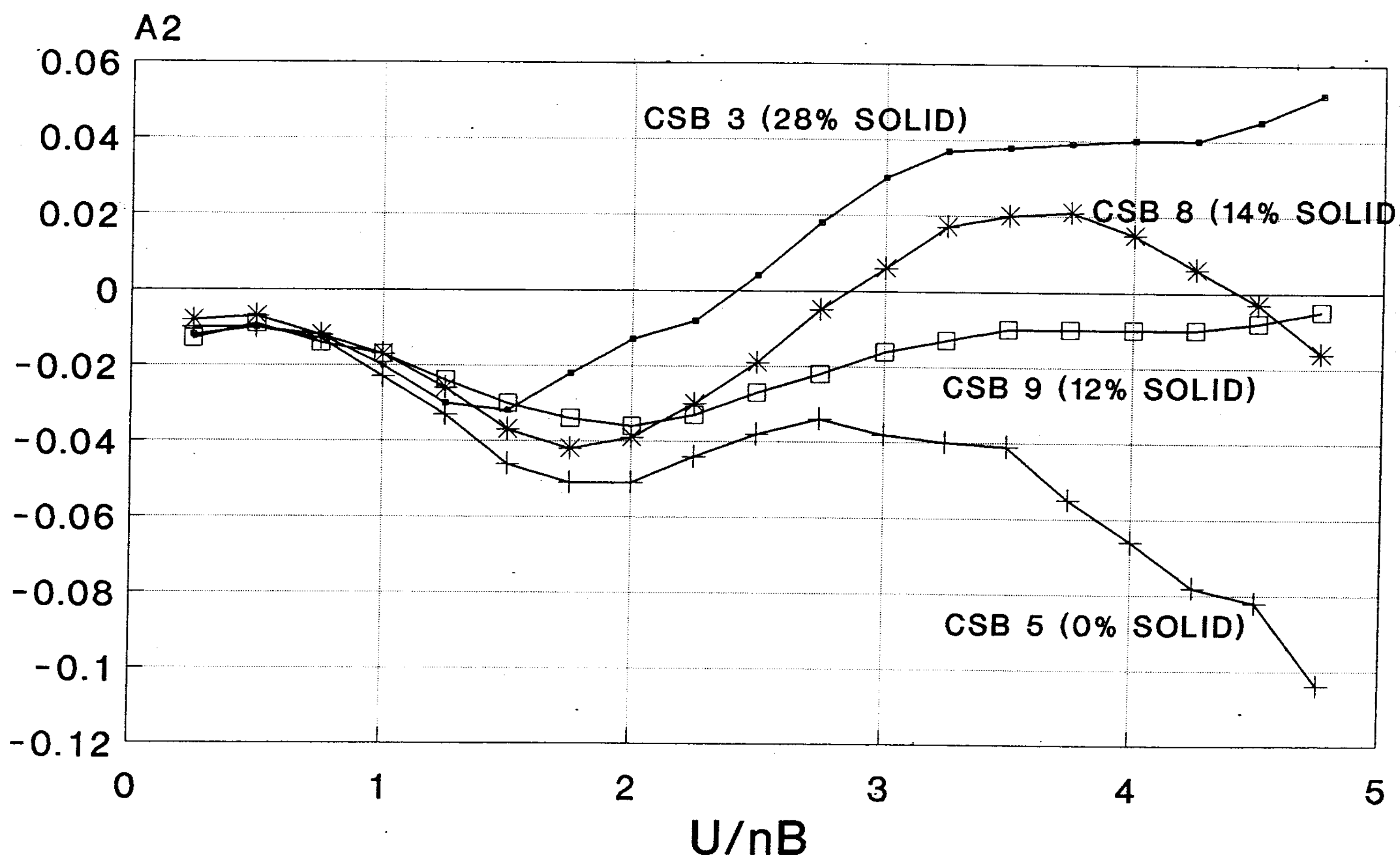


FIGURE S8

CSB 3, 5, 8, & 9

1/22/98



0 DEGREES

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FIGURE S9
A2 AS A FUNCTION OF PEDESTRIAN RAILING SOLIDS RATIO

REFERENCES

A. Scanlan, R. H. and Jones, N. P., *Wind Response Study Carquinez Strait Suspended Span*, Report for West Wind Laboratory, Inc. and OPAC Consulting Engineers, November 30, 1998.

B. Raggett, J. D., *Experimental Test Results Wind Study Third Carquinez Strait Bridge, Crockett to Vallejo, California,* West Wind Laboratory, Inc., November 1998.